Residual Clay Dams in the State of São Paulo, Brazil

Barrages en argile résiduelle dans l'Etat de São Paulo au Brésil

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SUMMARY

A description of seven earth dams constructed or under construction in the state of São Paulo, Brazil, is given in this paper. The borrow material for all these dams is residual soil from weathering of gneiss, basalt, or sandstone. A comparison of the physical properties, shearing strength, and permeability of the compacted residual soils used is presented. Problems that arise in the design and construction of the dams, related to the compaction of residual soils or foundations on decayed or fissured rocks are mentioned.

SOMMAIRE

Le présent article traite de la description de sept barrages en terre construits ou en construction dans l'Etat de São Paulo au Brésil. Les matériaux d'emprunt pour tous ces barrages sont les sols résiduels provenant de la décomposition du gneiss, basalte et grès. La comparaison des propriétés physiques, résistance au cisaillement et perméabilité après compactage des sols résiduels utilisés est présentée dans cet article. Sont également mentionnés, les problèmes surgis dans le projet et la construction des barrages concernant le compactage des sols résiduels et les fondations sur des roches fissurées ou décomposées.

ABOUT TEN YEARS AGO the government of the state of São Paulo planned the construction of a number of hydroelectric projects involving several concrete and earth dams. To take care of design, construction, and operation of these projects three state companies were organized: CHERP—Companhia Hidroelétrica do Rio Pardo; USELPA—Usinas Elétricas do Paranapanema; and CELUSA—Centrais Elétricas de Urubupungá. Three of the concrete gravity dams and five of the earth dams involved have already been completed, and three other earth dams are under construction.

The purpose of this paper is to describe the main design and construction features of these earth dams, where residual clayey soils were used extensively as borrow material, and where foundation problems peculiar to residual soils and weathered rocks had to be solved.

The cross-sections, nature of borrow materials, geological characteristics of foundations, and other data regarding these dams are shown in Fig. 1.

NATURE OF THE RESIDUAL SOILS USED AS BORROW MATERIALS

Residual soils in southern Brazil mainly originated from weathering of granitic, gneissic, or schistose rocks, basalt trap rocks, or sandstone beds. The profiles of those residual soils often show three main zones:

1. *An upper zone*, which is composed of a red or brownish sandy clay, called "porous" locally. It is a non-saturated soil with a macro-voided structure, very compressible, but with low moisture content. Weathering of gneissic and basaltic rocks produces a "porous clay," and weathering of sandstone produces a "porous sand" superficial layer.
2. *An intermediate zone*, composed of a sandy silt or clay, or even a clayey sand showing the original rock structure, occasionally with some fragments of solid rock. Sometimes, this zone is formed by a hard red layer in basalt soils or a compact clayey sand layer in sandstone soils.
3. *A lower zone*, which is very erratic in nature and is located just above partially weathered or fissured rock. It is composed of layers or boulders of weathered rock intermingled with thin layers of sandy-silty clays or soft completely decayed rock.

Materials for earth dams in southern Brazil are generally borrowed from the upper "porous" zone and less frequently from the intermediate zone. These materials are adequate for earth dam construction; they are easily excavated and when properly compacted exhibit low permeabilities and relatively high strengths.

The "porous" material from granite or gneiss is a sandy clay or a clayey sand, with 20 to 60 per cent of its grains smaller than 0.1 mm diameter and 5 to 50 per cent clay fraction (diam < 2 microns). Its liquid limits and plasticity indexes plot generally in the vicinity of the A-line in the plasticity chart (Fig. 2a). Sandstone soils are a mixture of a fine sand and 10 to 20 per cent of an active clay fraction. They often plot in the plasticity chart above the A-line and on the left side of the B-line (Fig. 2b), but when their liquid limit is higher than 50 per cent, they plot below the A-line. Basalt ("porous") material is a red clay with 20 to 60 per cent clay fraction (diam $< 2\ \mu$) and plots in the plasticity chart, generally above the A-line (Fig. 2c). Its liquid limits vary from 30 to 55 per cent. Less frequently, however, its liquid limit is as high as 75 per cent and plots below the A-line, mainly when its iron oxide content is high.

Grim and Bradley (1963), examining the clay mineral composition of such gneiss and sandstone "porous" soils, have found that their clay fraction is mainly kaolinite with a percentage of gibbsite, higher at the surface and disappearing at the bottom of the "porous" zone. In the deep residual sandstone soils, kaolinite is replaced by montmorillonite and related three-layer clay minerals. It seems that some kaolinite in the sandstone soils may be formed by way of an intermediate montmorillonite stage. This may explain the peculiar plot of these sandstone soils in the plasticity chart. At less

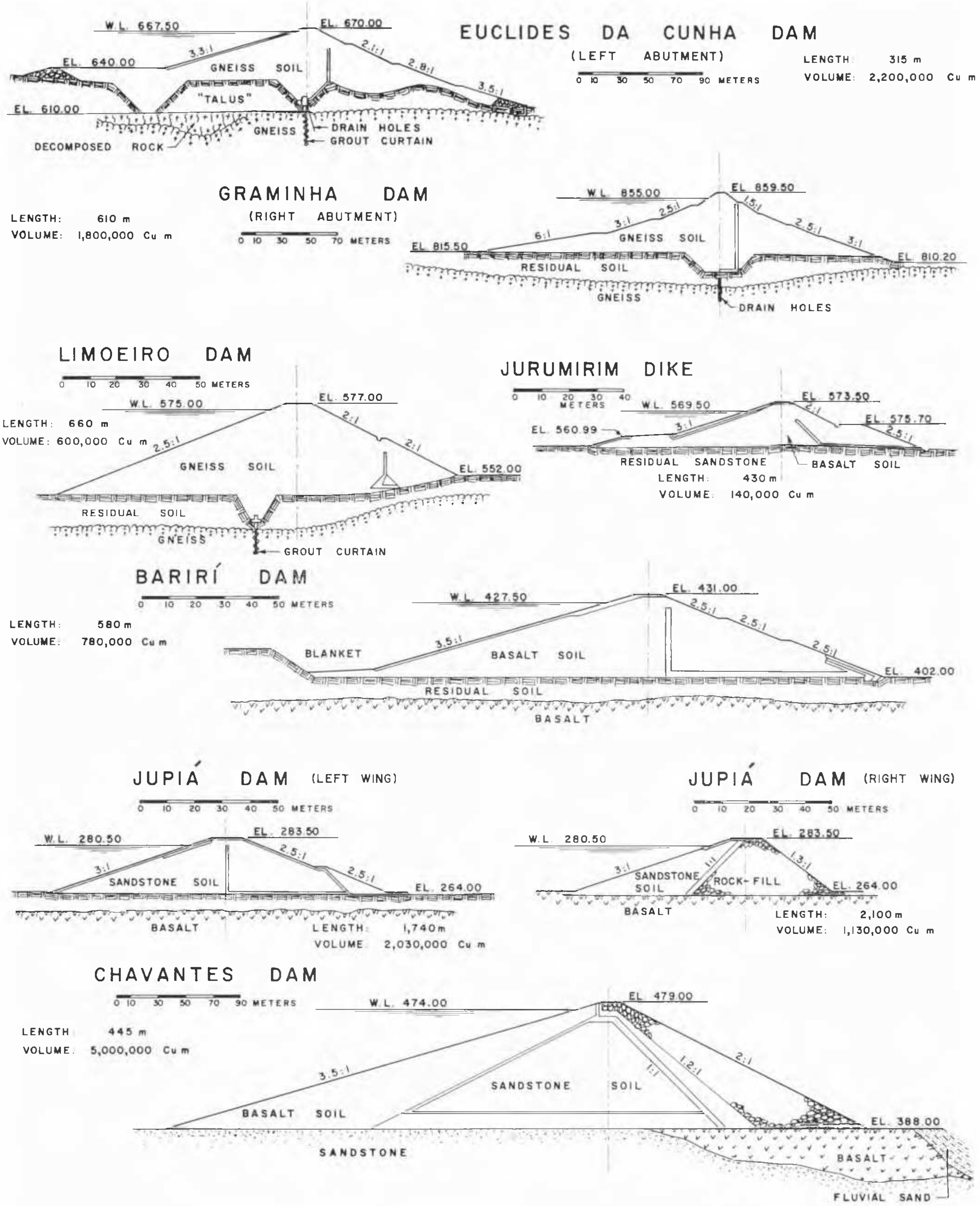

FIG. 1. Cross-sections of some residual clay dams.

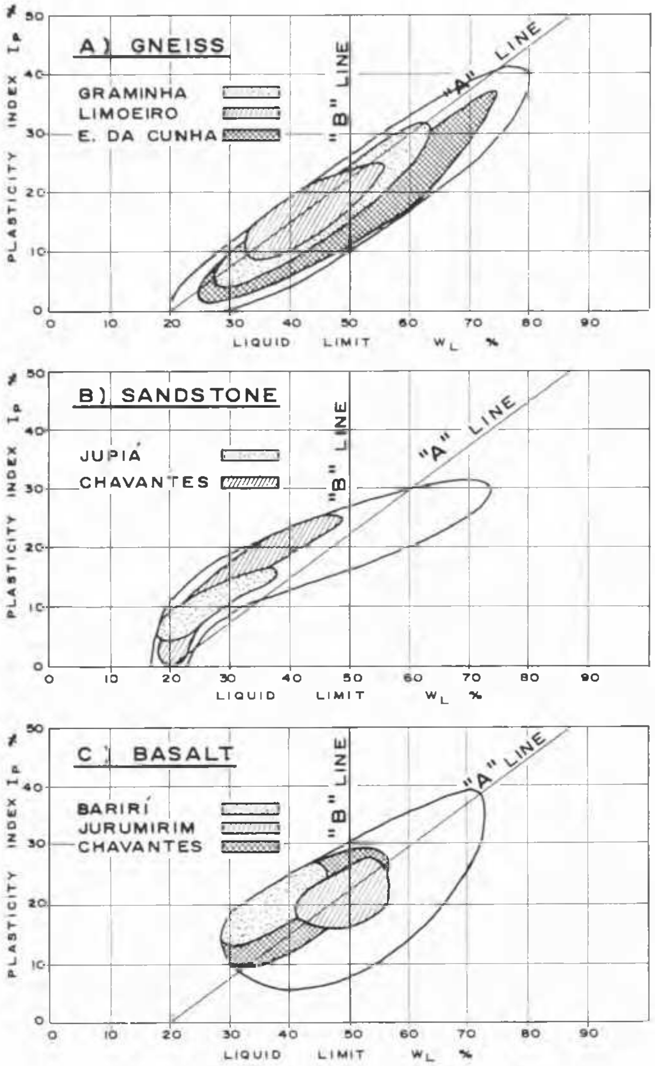

FIG. 2. Plots of Atterberg limits of residual soils in the plasticity chart.

advanced weathering stages the soils would have a small amount of a very active clay fraction plotting above the A-line; at more advanced stages of weathering the soils would, on the other hand, have a greater clay fraction content but the clay mineral present would be less active, plotting below the A-line.

"Porous" red clays from basalt, which are the famous fertile "terra roxa" of the coffee plantations, have not thus far been very well investigated as far as their clay mineral composition is concerned. A few determinations, however, have shown that kaolinite and iron oxide are their main products of weathering.

In general the "porous" uppermost zone of residual soils has a high content of iron oxides as a product of weathering. It is likely that its macro-voided structure is formed by lixiviation of iron oxides which precipitate to form the limonite crusts very commonly found at the bottom of such zones.

The natural moisture contents of all these soils are often very low: from 10 to 30 per cent. Only in rare instances do they become soaked by rain or capillary rise. Their natural densities are low, ranging from 1.4 to 1.7 grams/cu cm, and so their void ratios vary from 0.8 to 1.5.

The maximum dry densities of such materials, as compacted in the standard Proctor test, are generally low for the gneiss and basalt soils (ranging from 1.4 to 1.8 grams/cu cm) and high for sandstone soils (from 1.8 to 2.2 grams/cu cm). Fig. 3 shows compaction curves of residual materials used in the several dams described. Table I shows identification and compaction average characteristics of residual soils used as borrow materials in the seven earth dams. In this table columns corresponding to clay fraction and plasticity show maximum and minimum values of those characteristics, columns corresponding to compaction show average values and computed standard deviations from these values.

SHEARING RESISTANCE AND PERMEABILITY OF COMPACTED RESIDUAL SOILS

Teixeira da Cruz (1963) has shown that the shear strength parameters of compacted residual clays, in terms of effective stresses, are independent of the stress history, structure of the soils, and process of failure involved. On the other hand

TABLE I. AVERAGE CHARACTERISTICS OF BORROW MATERIAL

			Plasticity		Field compaction control					Shear parameters	Pore-pressure coefficient
Dam	Parent rock of borrow materials	Per cent clay (diam. <2μ)	w_L (per cent)	I_P (per cent)	w_f (per cent)	γ_d (grams/cu. cm.)	$w_f - w_0$* (per cent)	$\frac{\gamma_d \times 100}{\gamma_{d_{max}}}$† (per cent)	Permeability k (cm/sec)	Q tests: $\tau_f = c_\mu + \sigma \tan \phi$ effective stresses $\tau_p = c' + \sigma' \tan \phi'$	$m = \frac{u_f}{\sigma_1 - \sigma_{cr}}$
Limoeiro	Gneiss	8–40	32–55	10–25	21.±2.3	1.56±0.08	−1.1±2.4	96.3±3.2	$k_v = 4 \times 10^{-7}$ $k_h = 16 \times 10^{-7}$	$\tau_f = 0.6 + \sigma \tan 15°$ —	—
Euclides da Cunha	Gneiss	6–32	25–75	5–35	24.2±2.8	1.56±0.07	−0.8±0.9	101.0±2.2	$k_v = 10^{-7}$ $k_h = 3 \times 10^{-7}$	$\tau_f = 0.8 + \sigma \tan 15°$ $\tau_p = 0.4 + \sigma' \tan 26°$	0.55
Graminha	Gneiss	10–42	26–72	5–30	25.7±4.0	1.50±0.09	−0.4±0.5	101.5±2.0	$k = 6 \times 10^{-6}$	$\tau_f = 0.5 + \sigma \tan 14.5°$ $\tau_p = 0.2 + \sigma' \tan 29°$	0.64
Bariri	Basalt	10–40	28–45	12–28	18.9±4.2	1.71±0.11	−0.6±0.7	99.5±2.7	$k = 10^{-7}$	$\tau_f = 1.6 + \sigma \tan 19°$ $\tau_p = 0.4 + \sigma' \tan 29°$	0.50
Jurumirim	Basalt	6–26	42–56	15–27	18.4±5.5	1.72±0.11	−2.1±1.2	101.6±2.6	$k = 2 \times 10^{-7}$	— —	—
Jupiá (left wing)	Sandstone	8–22	16–38	5–18	10.6±0.9	1.96±0.04	−0.6±0.4	98.9±2.1	$k_v = 10^{-5}$ $k_h = 4 \times 10^{-5}$	$\tau_f = 0.8 + \sigma \tan 28°$ $\tau_p = 0.5 + \sigma' \tan 30°$	0.10
Chavantes	Basalt	19–47	28–58	9–29	23.5	1.59	—	—	—	$\tau_f = 0.8 + \sigma \tan 20°$ $\tau_p = 0.2 + \sigma' \tan 29°$	0.46
	Sandstone	6–20	16–48	0–25	11.9	1.89	—	—	—	$\tau_f = 0.5 + \sigma \tan 30°$ $\tau_p = 0.1 + \sigma' \tan 33°$	0.18

*w_f = field moisture content; w_0 = Proctor moisture content.

†γ_d = field dry density; $\gamma_{d_{max}}$ = Proctor dry density.

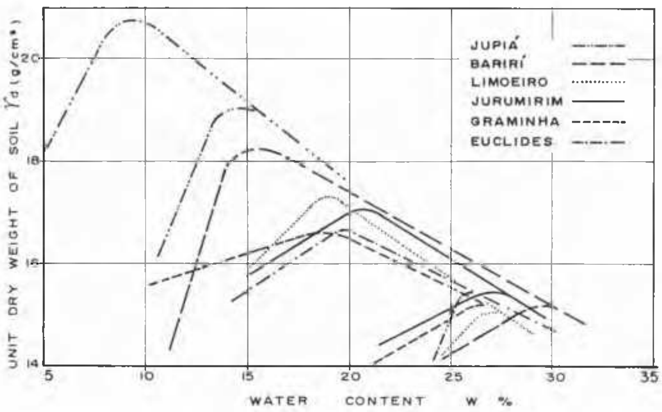

FIG. 3. Limiting compaction curves of residual soils.

Cruz states that "the behavior of a compacted residual clay when submitted to shear is very characteristic . . . and differs in many ways from the behavior of the same clay submitted to a different stress history prior to the same shear process."

Actually, during design and construction of some of the dams under discussion, it has been confirmed that the average values of cohesion and angle of friction, in terms of effective stresses, are the same regardless of the method of compaction (test specimens compacted in the laboratory or trimmed from undisturbed samples of the compacted fill). On the other hand results of tests in terms of total stresses are quite different in both cases, which shows that the development of pore pressures is different for each case.

Measurement of pore pressures, u_f, developed at a rupture, in quick tests, have shown that u_f, in general increases linearly with chamber total pressure σ_3, according to the expression:

$$u_f = m(\sigma_3 - \sigma_{cr})$$

where σ_{cr} is a confining pressure corresponding to a transition from positive to negative pore pressures and m is a constant; both m and σ_{cr} depend on type of soil and compaction conditions. For high values of σ_3 it is possible to assume u_f simply proportional to σ_3 in the design of residual clay dams.

Table I shows results of shear parameter determinations for some residual soil samples, compacted at optimum moisture content and Proctor maximum density. It also shows the values of the pore-pressure coefficient m, obtained with tests on laboratory compacted samples, for values of σ_3 up to 10 kg/sq.cm.

Permeability tests with a low (about 1 m) constant head permeameter were run on samples compacted at optimum moisture content and maximum density. In some cases these tests were run normally and horizontally to the compaction plane. Table I shows average results of permeability coefficients for the residual soils used as borrow material in the several dams under consideration.

DESIGN AND CONSTRUCTION OF RESIDUAL SOIL EARTH DAMS

The rational design of earth dams using the kinds of residual soils described here was introduced in Brazil in 1948 by Karl Terzaghi, in connection with the Vigario Dam and Dike construction in the state of Rio de Janeiro. Both the dam and the dike have homogeneous sections with a vertical sand filter just downstream of their axes, a feature which has been incorporated in many of the major earth dams designed for southern Brazil.

As compacted residual soils are highly impervious and have sufficient strength, the adoption of homogeneous sections in the design of residual soil dams is most convenient, both for construction and design computation reasons, although the residual soils themselves are very non-homogeneous. In fact, even a soil derived from a complete chemical deterioration of a uniform rock, appearing quite homogeneous, may show a wide variation of properties determined by tests performed on samples taken from various points. On the other hand, even with soils non-homogeneous to visual inspection, it is almost impossible to attempt any zoning in the dam section, because of the erratic nature of the variation of its properties. However, fortunately, this wide variation results in a uniform scattering of the soil characteristics in all points of the soil mass, and so the section can be considered statistically homogeneous.

This large range of properties found in residual soils is reflected in their compaction characteristics and, consequently, in field compaction control. A rather inaccurate and time-consuming compaction control method based on families of Proctor curves initially developed to take care of these variations has now been abandoned in favour of development of Hilf's method.

Degrees of compaction exceeding 95 per cent of standard Proctor maximum and moisture contents ranging from optimum to 2 per cent below optimum are generally specified for Brazilian dams. For the design of most of the dams described here it was decided, in view of the wide scattering of values of cohesion and angles of shear resistance, to compute first, for several values of ϕ, the cohesion necessary to ensure stability under critical situations: that is, just after construction and after rapid drawdown. These values were plotted as angle of friction *versus* necessary cohesion. Then the several pairs of ϕ and c values obtained in quick and consolidated quick tests were plotted in the same graph. If all values plotted to the right of the ϕ *versus* c (necessary) curves, the section was considered stable. If a few of the test results plotted very near or to the left of the curves, then an investigation of the possibility of avoiding the use of the soils corresponding to such values, was carried out. In many cases it was found that those soils could be detected by visual inspection or by some visible characteristic, for instance a high mica content.

Regarding the problem of piping, it was observed that the residual soils here described are, in general, very resistant to piping produced by excessive seepage forces. However, it is of major concern in the design of such dams to avoid piping caused by cracking of the compacted material. It was observed that the usual placement of the soil, at moisture contents somewhat below optimum, which often coincides with the natural moisture content of the soils in the borrow pits, frequently leads to serious cracking in the embankment, due to differential settlement. This phenomenon is probably due to the fact that the natural water contents of residual soils in the superficial layers is almost at the plastic limit in southern Brazil.

The use of a vertical sand filter in those dams is used to remedy the adverse effects of cracking. The vertical filter will act as a barrier to seepage along such cracks. Curving the axis of the dam is also adopted, in high dams, as an additional measure against cracking.

FOUNDATIONS OF DAMS ON RESIDUAL SOILS AND DECOMPOSED ROCKS

In the great majority of the cases, dams in southern Brazil have their central sections corresponding to the river beds, founded on sound or fissured rock, although often the abutments are laid on residual soils or weathered rocks. This is the case in the Euclides da Cunha, Graminha, and Limoeiro dams (Fig. 1). As a rule, in such cases a cut-off is likely to be considered. A large trench is excavated through the upper and intermediate zones of the residual soils to a depth where the occurrence of boulders or hard layers of partially weathered rock make continued excavation difficult. However, from this level down to the sound rock, there is a layer of a more or less deteriorated rock, eventually intermingled with seams or layers of residual soils, which are generally permeable.

To take care of this eventuality three procedures are used. The first is to construct a positive cut-off to sound rock below the trench, with a narrow cut, back-filled with concrete to form an impervious concrete wall which also serves as a grouting cap. Finally, grouting of the fissured rock underneath to form an impervious grout curtain is carried out. This was done in the Euclides da Cunha and Limoeiro dams (Fig. 1). The second procedure is to drill drainage holes from the bottom of a trench excavated in the soft material. These drains are connected to a filter, as in the Graminha Dam (Fig. 1). The third procedure is to provide the dam with a horizontal sand filter, downstream of its centre line, laid directly on the pervious strata in order to collect the seepage through the foundations, as in the Jurumirim dike, and Bariri Dam (Fig. 1).

A series of piezometers were installed in each of the above-mentioned dams to observe seepage gradients in the foundations. Although sufficient information is not yet available, it is possible to foresee that the adoption of positive cut-offs in the design of the dams did not noticeably improve their safety as far as underseepage gradients are concerned. It seems, as stated by Casagrande (1961), that in normal cases the presence of a positive cut-off and a single-line grout curtain does not modify the piezometric surface of the water flowing through the foundations.

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